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# SURCHARGE FILL AND VERTICAL DRAIN SYSTEM IMPROVES SOFT CLAY SITE

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**ABSTRACT**

The Intermountain Power Project in Delta, Utah required construction of a Railcar Service Center near Provo, Utah. Preliminary investigations related to the design of the proposed facility revealed the presence of soft clays at the site adjacent to the foothills of the Wasatch Mountains.

Detailed geotechnical laboratory and field investigations were performed using relatively undisturbed California samples, thin-walled Shelby tube samples, and cone penetration test soundings. The detailed investigations revealed the presence of a significant layer of deep, soft varved clay. This soft clay layer originated from alluvial deposits from the old Lake Bonneville.

Because of the need to develop the site to substantially higher final grade elevation, which in turn necessitated the placement of supplemental fill material, the project team decided to use a preload/surcharge fill (comprised of soil to be used in the ultimate site development) and prefabricated vertical drain system designed to mitigate long term settlements for the facility. The impetus for adopting this design approach was to use shallow foundation systems for the project, as opposed to a more expensive foundation system consisting of driven pile foundations. The designers used two basic approaches to the ultimate development of the site as impacted by the underlying soft, compressible varved clay.

1. Laboratory triaxial strength testing and cone penetration test (CPT) sounding were used to develop a site-specific shear strength profile.
2. One-dimensional consolidation testing, including time rate analyses, was used to define a past pre-consolidation confining pressures and the anticipated virgin compression parameters.

The overall preload system design accounted for the rate of fill installation, global stability of the fill/natural soil profile, and anticipated internal drainage conditions of the varved clay. Design predictions related to magnitude, rate and duration of the preload settlement are compared with actual settlement measurements of the fill and deeper, intermediate strata within the varved clay. The magnitude of settlement related to the surcharge fill approached 7 inches.

An ancillary and coincident activity related to the preload activities included the performance and interpretation of a pile load test installed into the varved clay profile. The strength profile developed for project design is also used to analyze the load transfer mechanisms during the pile load test, which also provided an understanding of the ultimate pile capacity as defined by the load test.

**Key Words:** preload, surcharge, vertical drains, varved clay, consolidation settlement

## INTRODUCTION

The Intermountain Power Project (IPP) was designed and built from 1980 through 1986 for the Los Angeles Department of Water and Power (LADWP). The project was a coal-fired generating station consisting of two pulverized coal units. The net rated generating capacity of the station is 1,650 MW. Coal for the power plant is transported from the Powder River Basin to the plant site near Delta, Utah by train. The project also required construction of a Railcar Service Center in Springville, Utah. The Railcar Service Center would serve as a maintenance facility for railcars used to transport coal. The project location is shown on Figure 1.

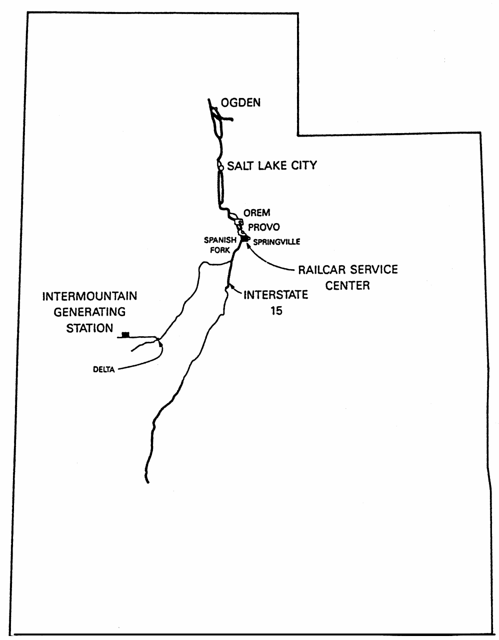



*Fig. 1. Site Location*

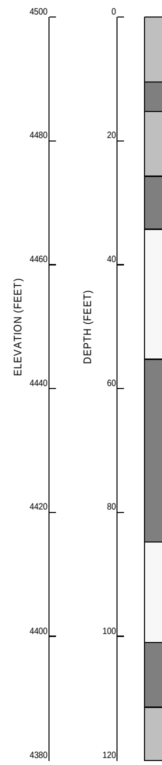
AREA GEOLOGY

The Railcar Service Center site lies within the Basin and Range Physiographic Province. The soils were deposited in the former Lake Bonneville which covered nearly 20,000 square miles of western Utah and parts of eastern Nevada and southern Idaho.

Deposits from lakes are typically fine-grained soils. Sands are deposited near the shore in areas of melt water flows. Silt and clay are deposited during warm periods when the water is more turbulent. During cold periods when melt water and inflow cease, the finer clay fraction still in suspension continues to settle. Lacustrine deposits are typically soft, compressible, and sensitive if they have not been exposed to desiccation. The deposits display anisotropic strength and permeability properties.

GROUND CONDITIONS

The grade at the site is approximately elevation 4,500 feet (MSL). The soils at the site consist mainly of interbedded silts and silt clay, with occasional layers of silty sand. The cohesive soils are generally soft to firm, low plasticity, and overconsolidated. Figure 2 illustrates a typical soil profile at the Railcar Service Center Building.




| Description | w | $w_P$ | $w_L$ | PI | DUW | $C'_c$ | $C'_r$ | $S_u$ | Phi |
|---|---|---|---|---|---|---|---|---|---|
| Silty CLAY (CL), Sandy SILT (ML), and Silty SAND (SM) | 36 | 13 | 29 | 16 | 88 | 0.17 | 0.015 | 700 | |
| Silty CLAY (CL) | 34 | 18 | 33 | 15 | 86 | 0.093 | 0.012 | 100 | |
| Silty SAND (SM), Sandy SILT (ML), and Silty CLAY (CL) | 45 | 18 | 44 | 26 | 70 | 0.082 | 0.016 | 700 | |
| Silty CLAY (CL) | 33 | 24 | 52 | 28 | 89 | | | 1100 | |
| Silty SAND (SM) | 25 | | | | 99 | | | | 36 |
| Silty CLAY (CL) | 43 | 21 | 45 | 24 | 79 | | | 1100 | |
| Sandy SILT (ML) and Silty Sand (SM) | | | | | | | | | |
| Silty CLAY (CL) | | | | | | | | | |
| Sandy SILT (ML) | | | | | | | | | |

*Fig. 2. Typical Soil Profile*

Results of the consolidation tests on the cohesive materials revealed past consolidation pressures which were at least 1.7 ksf greater than the present effective overburden pressure as shown on Figure 3.

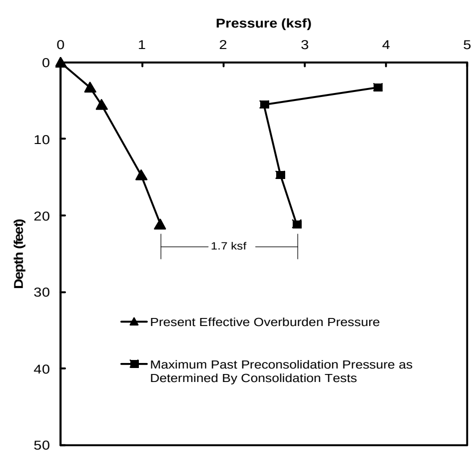



*Fig. 3. Preconsolidation Pressure Plot*

Undrained shear strengths were estimated using unconfined compression testing performed on ring-lined split barrel samples, CPT soundings, and pocket penetrometer testing. Figure 4 provides the undrained shear strength values obtained from the investigation and the parameters used in design.

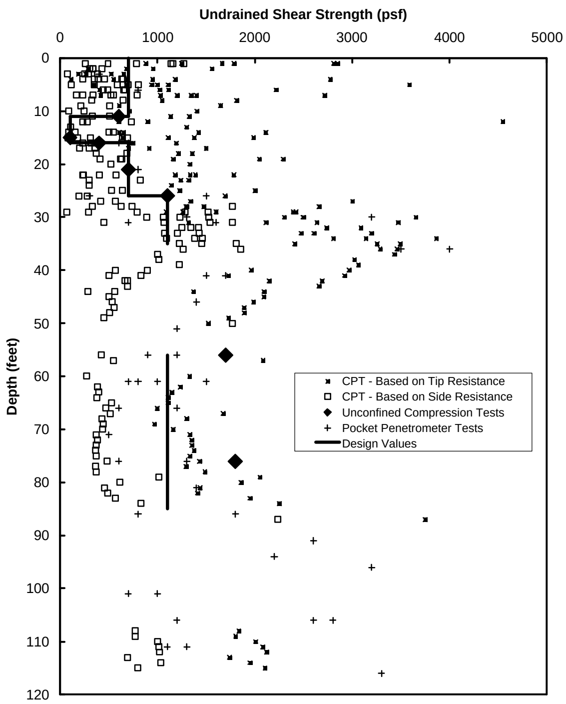



*Fig. 4. Undrained Shear Strength Plot*

PRELOAD DESIGN

Shallow foundations were investigated to determine if bearing capacity and settlement requirements could be met. The allowable bearing capacity for square and strip footings was estimated to be approximately 1,500 psf and 1,200 psf, respectively. The settlement of shallow footings with these contact pressures was estimated and found to be about 1.5 inch. Since the contact pressures were less than the 1.7 ksf overconsolidation pressure, the settlements were due entirely to recompression of the clays.

In addition to the settlement of the shallow foundations, the settlement due to site fill placement was estimated. Site development plans indicated that 6 to 9 feet of fill were to be placed at various locations at the site. The estimate settlements from this fill ranged from about 2 to 4 inches.

Given the estimated excessive settlements, the project team decided to consider preloading the site with and without vertical drainage. Settlement beneath the footing foundations at the Railcar Service Center Building would occur mostly in the silt and clay layers above Elevation 4,465 feet (see Figure 3). To be effective the preload had to affect this soil zone to a contact pressure approaching that of the building foundations. The required preload was determined to be 16 feet of fill, 6 feet of site fill plus 10 additional feet of fill.

The rate of consolidation under this preload was estimated using consolidation theory with instantaneous loading. Without supplemental vertical drainage elements, approximately 20 months would be required to achieve 90 percent consolidation in the silt and clay above Elevation 4,465 feet. The amount of time required to achieve 90 percent consolidation could be significantly reduced by using uniformly spaced vertical drainage elements (see Figure 5). It was understood that the time rate of settlement was also highly dependent on the spacing and degree of continuity of the more pervious horizontal drainage layers within the compressible strata.

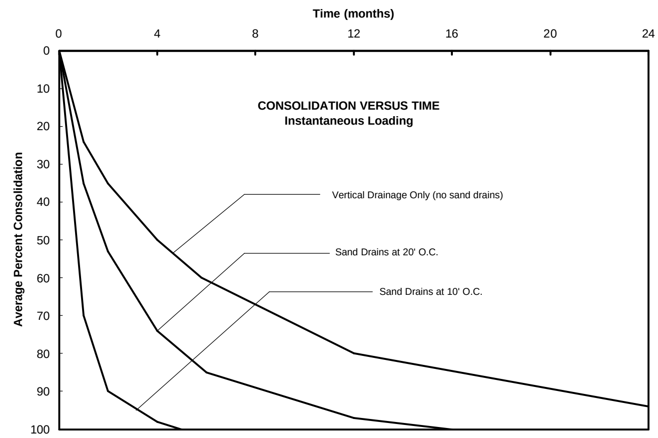



*Fig. 5. Estimated Settlement Plot with Different Drain Spacings*

The preload fill could not be constructed too rapidly over a soft clay where a rotational slide or base failure could occur due to the development of excessive pore water pressures. To avoid such failures, movement devices, both horizontal and vertical, and piezometers were planned to monitor the settlement of the soil surface supporting the surcharge, the pore water pressures in the subsoil, and the heave or lateral movement of the natural ground beyond the limits of the preload fill.

PRELOAD RESULTS

Approximately 1,600 vertical prefabricated wick drains were installed over the area of the subgrade that would experience compression from the service center mat. The drains were installed to the bottom of the compressible silty clay layer. The vertical drains were terminated at the ground surface in a two foot layer of well-graded sand which formed a drainage medium for the water flow. The sand was then covered with 16 feet of fill and left to consolidate.

About 3 months after fill placement began, approximately 7 inches of ground surface settlement had occurred as indicated on Figure 6. The design prediction for 100 percent consolidation of the clays was also about 7 inches. The estimated time to achieve 100 percent consolidation was approximately 5 months, as compared to the actual time of 3 months to achieve 100 percent consolidation. However, the presence of horizontal drainage layers in the varved clay apparently accelerated the consolidation process. Pore water pressures in the varved clay typically built up and dissipated quickly after loading. A typical pore water pressure response from a piezometer is indicated on Figure 7.

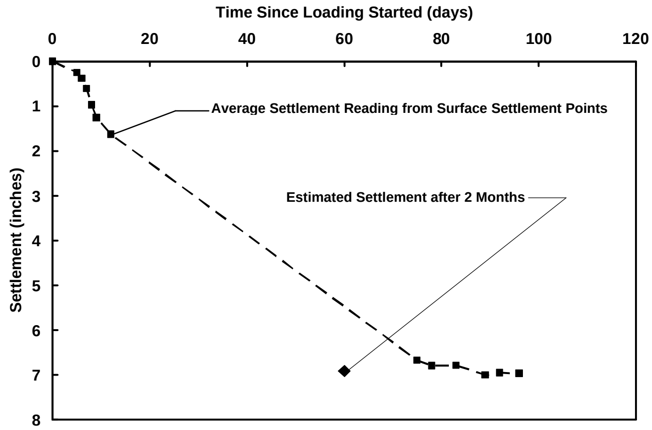



*Fig. 6. Predicted Versus Actual Settlement Plot*

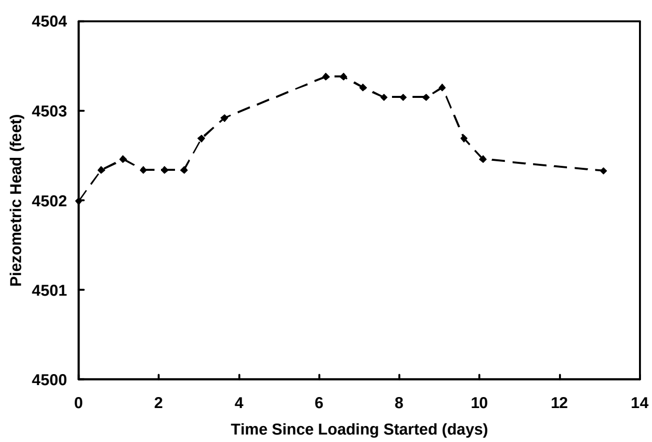
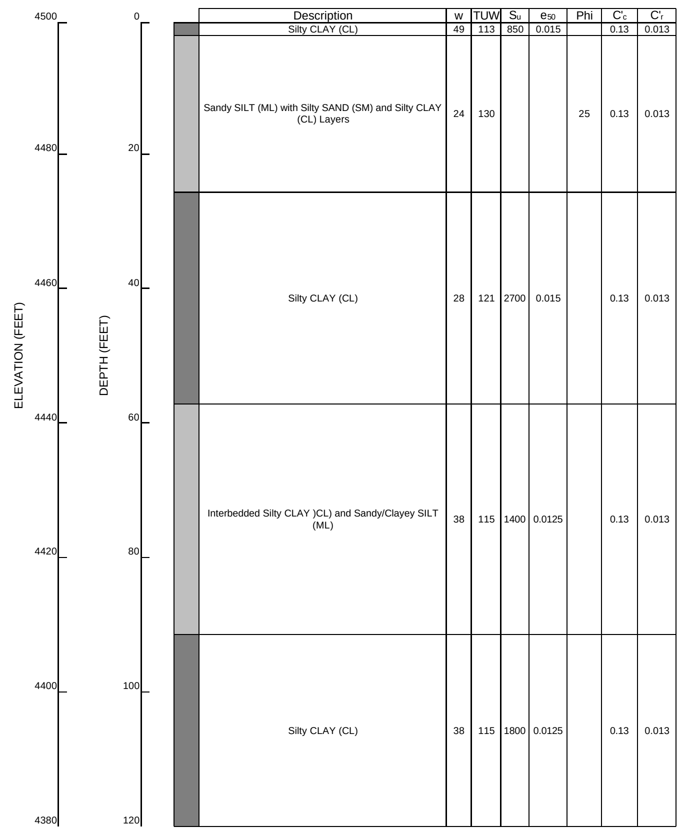



*Fig. 7. Typical Pore Pressure Plot*

PILE DESIGN

Because extensive field and laboratory testing efforts were undertaken to define the soil profile in support of design activities associated with the preload/surcharge fill, an unexpected benefit to the project occurred. The project team ultimately decided to support the Hobble Creek Bridge on a deep foundation system comprised of cast-in-place, step taper piles. The development of a design soil profile at the Hobble Creek Bridge, based in part on the efforts required to design the surcharge fill system, enabled the project team to develop a timely and appropriate, if not slightly conservative, pile design.

The ultimate pile design consisted of an estimated pile length of 75 feet using a “Raymond” Step Taper pile system. This pile system consisted of incremental casing lengths of 8 feet with diameters ranging from 8.5 inches at the pile tip to 15.5 inches at the pile top. This type of pile is generally driven using an internal mandrel system. Based on the design soil profile developed for the Hobble Creek location (see Figure 8), and using a pile design method generally predicated on developing the majority of the pile capacity from side friction in the underlying silt and clay soils, an ultimate pile capacity of approximately 190 kips (95 tons) was calculated.




| Description | w | TUW | $S_u$ | $e_{50}$ | Phi | $C'_c$ | $C'_r$ |
|---|---|---|---|---|---|---|---|
| Silty CLAY (CL) | 49 | 113 | 850 | 0.015 | | 0.13 | 0.013 |
| Sandy SILT (ML) with Silty SAND (SM) and Silty CLAY (CL) Layers | 24 | 130 | | | 25 | 0.13 | 0.013 |
| Silty CLAY (CL) | 28 | 121 | 2700 | 0.015 | | 0.13 | 0.013 |
| Interbedded Silty CLAY )CL) and Sandy/Clayey SILT (ML) | 38 | 115 | 1400 | 0.0125 | | 0.13 | 0.013 |
| Silty CLAY (CL) | 38 | 115 | 1800 | 0.0125 | | 0.13 | 0.013 |

*Fig. 8. Hobble Creek Soil Profile*

The actual installed pile system consisted of a step taper system consisting of incremental casing lengths of 12 feet each with pile diameters ranging from 9.5 inches at the pile tip to 15.5 inches at the pile top. The final installed length generally ranged approached 82 to 83 feet. During the field driving activities, two compressive load tests were conducted on individual piles located beneath the north abutment of the Hobble Creek Bridge. Figures 9 and 10 depict the plots of deformation versus vertical load for piles numbered NA-24 and NA-13. Please note that the telltale was extended to the pile tip for the load test conducted at Pile No. NA-24 to observed tip movement as related to applied load at the top of the pile.

The pile load tests were conducted to twice the anticipated design capacity of 32.5 tons (65 kips), resulting in a test load of 65 tons. Vertical deformation at 65 tons of applied load ranged from approximately 0.115 inch for Pile NA-13 to 0.175 inch for Pile NA-24. These deformations were significantly less than the calculated elastic shortening of the given pile system, implying that each pile could likely support a significant increment of additional load before approaching failure. The minimal amount of tip movement revealed by the telltale for Pile NA-24 further indicated that the majority of the 65-ton test load was being supported by side friction within the overlying silt and clay layers.

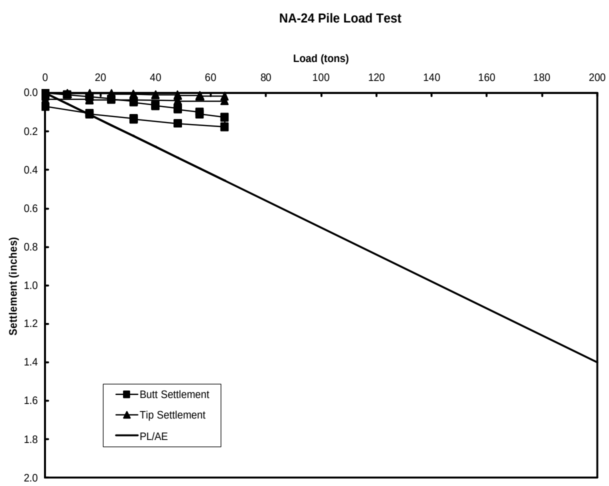



*Fig. 9. Pile Load Test Plot – NA-24*

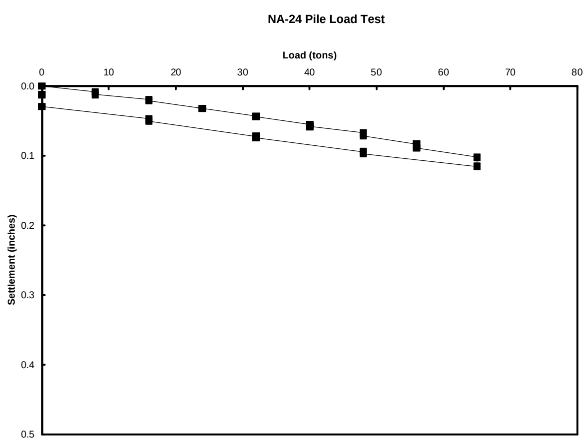



*Fig. 10. Pile Load Test Plot – NA-13*

CONCLUSIONS

Designers used consolidation testing to accurately predict the consolidation settlement of soft, varved clay layers. The rate of excess pore water pressure dissipation was enhanced with the use of vertical drains. The rate of dissipation was faster than anticipated due to the interlayered nature of the compressible silty clays. The preload system effectively reduced long term settlements of the service center mat foundation. The service center is effectively operating today.


*Fig. 11. Railcar Service Center Today*

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